



Bureau of Mines Report of Investigations/1984

Field Measurement of Rock Displacement and Support Pressure at 5,955-Ft Level During Sinking of Deep Circular Shaft in Northern Idaho

By M. J. Beus and M. P. Board



UNITED STATES DEPARTMENT OF THE INTERIOR

Report of Investigations 8909

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UNITED STATES DEPARTMENT OF THE INTERIOR

William P. Clark, Secretary

BUREAU OF MINES

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Library of Congress Cataloging in Publication Data:

Beus, Michael J.

Field measurement of rock displacement and support pressure at 5,955-ft level during sinking of deep circular shaft in northern Idaho.

(Report of investigations / United States Department of the Interior, Bureau of Mines ; 8909)

Includes bibliographical references.

Supt. of Docs. no.: I 28.23:8909.

1. Ground control (Mining). 2. Rock deformation. 3. Rock mechanics. 4. Silver mines and mining--Idaho--Shoshone County. I. Board, M. P. (Mark P.). II. Title. III. Series: Report of investigations (United States. Bureau of Mines) ; 8909.

TN23.U43 [TN288] 622s [622'.25] 84-600 192

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UNIT OF MEASURE ABBREVIATIONS USED IN THIS REPORT

ft	foot	in/in	inch per inch
ft/d	foot per day	pct	percent
h	hour	psi	pound per square inch
in	inch		

FIELD MEASUREMENT OF ROCK DISPLACEMENT AND SUPPORT PRESSURE AT 5,955-FT LEVEL DURING SINKING OF DEEP CIRCULAR SHAFT IN NORTHERN IDAHO

By M. J. Beus¹ and M. P. Board²

ABSTRACT

The Bureau of Mines, under a cooperative agreement with Hecla Mining Co., is conducting a rock mechanics study of the 6,200-ft-deep circular, concrete-lined Silver Shaft in northern Idaho. The objective of the project was to instrument and evaluate lining behavior and rock deformation when a deep shaft is constructed in a high in situ stress field and anisotropic rock mass. Rock mass displacements were monitored with multiple-position borehole extensometers. Liner stress and strain were measured with concrete pressure cells and embedment strain gauges.

Field data from the 5,955-ft test level show that rock displacement is controlled by rock mass anisotropy. Liner stress and strain levels reflect the high in situ stress field as well as nearby station and raise construction. Maximum stresses measured in the lining, as a result of shaft excavation, are about one-third of its compressive strength. Subsequent loading pocket excavation resulted in significantly increased stress and strain levels in the liner. Continued monitoring and preliminary analysis of the data indicate that concrete-lined, circular shafts are stable for the high, unequal horizontal stress fields and rock anisotropy encountered in deep-level mining. Adjacent development work can be expected to significantly alter the initial stress and displacement conditions.

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INTRODUCTION

For many years, the Bureau of Mines has conducted research to determine the in situ stress field and to develop structural guidelines for designing shafts in deep vein-type mines. Much of this work has centered on determining the in situ conditions that affect the structural stability of shafts in the Coeur d'Alene District of northern Idaho.³ Annual shaft repair and maintenance costs resulting from high rock stresses and excessive displacement cause damage to timber and steel shaft sets in the millions of dollars per year.

In an effort to eliminate the ever-increasing repair and maintenance costs,

Hecla Mining Co. is further developing its Lucky Friday Mine near Mullan, ID, with completion of an 18-ft-diam, 6,200-ft-deep circular, concrete-lined shaft. A cooperative agreement was established between the Bureau and Hecla early in 1980 to instrument and evaluate the structural response of the new shaft during and after construction. Test levels were installed at 2,414, 4,063, 5,191, and 5,955 ft. This report presents results and preliminary analysis of data from the 5,955-ft level. Follow-up reports will compare results from all four test levels and present a detailed analyses.

ACKNOWLEDGMENTS

The authors express their gratitude to officials and personnel of Hecla Mining Co., particularly P. Boyko, project manager; C. Flood, and T. Stephenson, mining technicians; and J. S. Redpath Co., the shaft sinking contractor, for assistance with installation of the instruments.

Further thanks goes to R. Muhs, mining engineer; J. K. Whyatt, mining engineer;

J. R. McVey, supervisory electronics technician; all of the Bureau's Spokane Research Center (SRC); and E. Hardin, geophysicist, of Science Applications, Inc., for computer hardware and software development, data processing, and assistance with instrument preparation and installation.

SITE DESCRIPTION

The Lucky Friday Mine is located at the eastern end of the Coeur d'Alene mining district near Mullan, ID (fig. 1). The mine lies in close proximity to the Osburn Fault, which is the most dominant structural feature of the district, striking west-northwest with several miles of lateral displacement.

Rocks in the Lucky Friday Mine area are mainly argillitic quartzites with varying amounts of interbedded argillaceous material. Most rocks belong to the St. Regis and Revett Formations of Precambrian age. The mineral deposits occur as steeply dipping veins, offset by many small

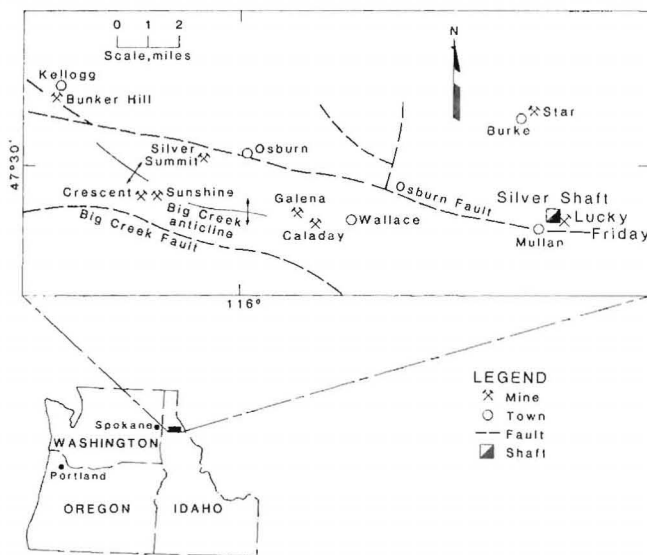


FIGURE 1. - Coeur d'Alene mining district showing location of Lucky Friday Mine and Silver Shaft.

³Beus, M. J., and S. S. M. Chan. Shaft Design in the Coeur d'Alene Mining District, Idaho--Results of In Situ Stress and Physical Property Measurements. Bu-Mines RI 8435, 1980, 39 pp.

faults. The main minerals are silver-bearing tetrahedrite and galena.

The Silver Shaft is located about 1,000 ft west of the currently used No. 2 shaft. Conventional drill-and-blast sinking practices were followed using handheld sinkers, bench blasting, and a cactus grab rotary-boom mucker mounted on the bottom deck of the Galloway stage.

INSTRUMENT INSTALLATION

Design of the shaft measurement system, including orientation and location of the instruments, is based on the in situ stress and rock mass conditions and the harsh shaft environment. Details of the design and selection of instrumentation and data acquisition will be described in a separate report.

The installation sequence followed a similar procedure for each level and required about 2 days. Geologic mapping was followed by locating and drilling multiple-position borehole extensometers (MPBX) holes within 4 ft of the shaft bottom. Installation of the pressure cells (PC's) and strain gauges (SG's) occurred at the same horizon once the concrete forms reached this level. Installation was designed to minimize interference with shaft sinking and was normally scheduled during holidays and maintenance periods. Figure 2 shows a section and plan view of the 5,955-ft level test site.

Drilling of the MPBX holes was by far the most time-consuming and difficult task. The holes were located as close to the shaft bottom as possible and oriented normal, perpendicular, and 45° to the steeply dipping bedding planes. Hole depth was 55 to 60 ft to provide for a zero-displacement-reference anchor position. Drilling time for the 150 ft of 3-in hole, using an air-track mounted drifter, totaled 19 h. Small "dogholes" were blasted at each hole location so that the protective blackout over the MPBX head could be recessed flush with the shaft walls.

Shaft steel and services were advanced concurrent with sinking. Initial support at the face consisted of 5-ft split sets over wire mesh. The 1-ft nominal thickness lining was maintained about 30 ft above the shaft bottom in 15-ft advances. The shaft-sinking contractor achieved overall average advance rates of 10 ft/d with a maximum 1-month record of 473 ft.⁴

After completion of the installation holes at the selected depth and orientation, the MPBX bundle was carried from the assembly site to the shaft collar, slung below the sinking can, and lowered down the shaft. This assembly was then guided into the hole as the can was lowered. The MPBX head was positioned in the doghole and the anchoring sequence initiated. Figure 3A illustrates the procedure.

The flex conduit housing the MPBX signal cable was tied to the wire mesh on the shaft walls and routed to a 4-in-diam pipe cast into the concrete liner, which exited at the junction box (J-box). The section of conduit left exposed between the MPBX head and the pipe was covered with bull hose, and the MPBX head was covered with 1/4-in steel plate bolted to the shaft walls for further protection from blasting. Sinking began as soon as connection was made to the J-box. Average total MPBX installation time was 9 h.

After the shaft bottom advanced 17 ft beyond the MPBX level, the Galloway and 3-ft curb ring were lowered for the liner instrumentation pour, as illustrated in figure 3B. The curb ring was positioned as the PC's and SG's were transported down the shaft. A horizontal profile of the shaft walls was obtained, using the

⁴McKinstry, B. A. Deepest Single Lift Shaft in North America Near Completion. Paper in Proceedings of the Rapid Excavation and Tunneling Conference (Chicago, IL, June 12-16, 1981). Soc. Min. Eng. AIME, Littleton, CO, 1981, pp. 493-512.

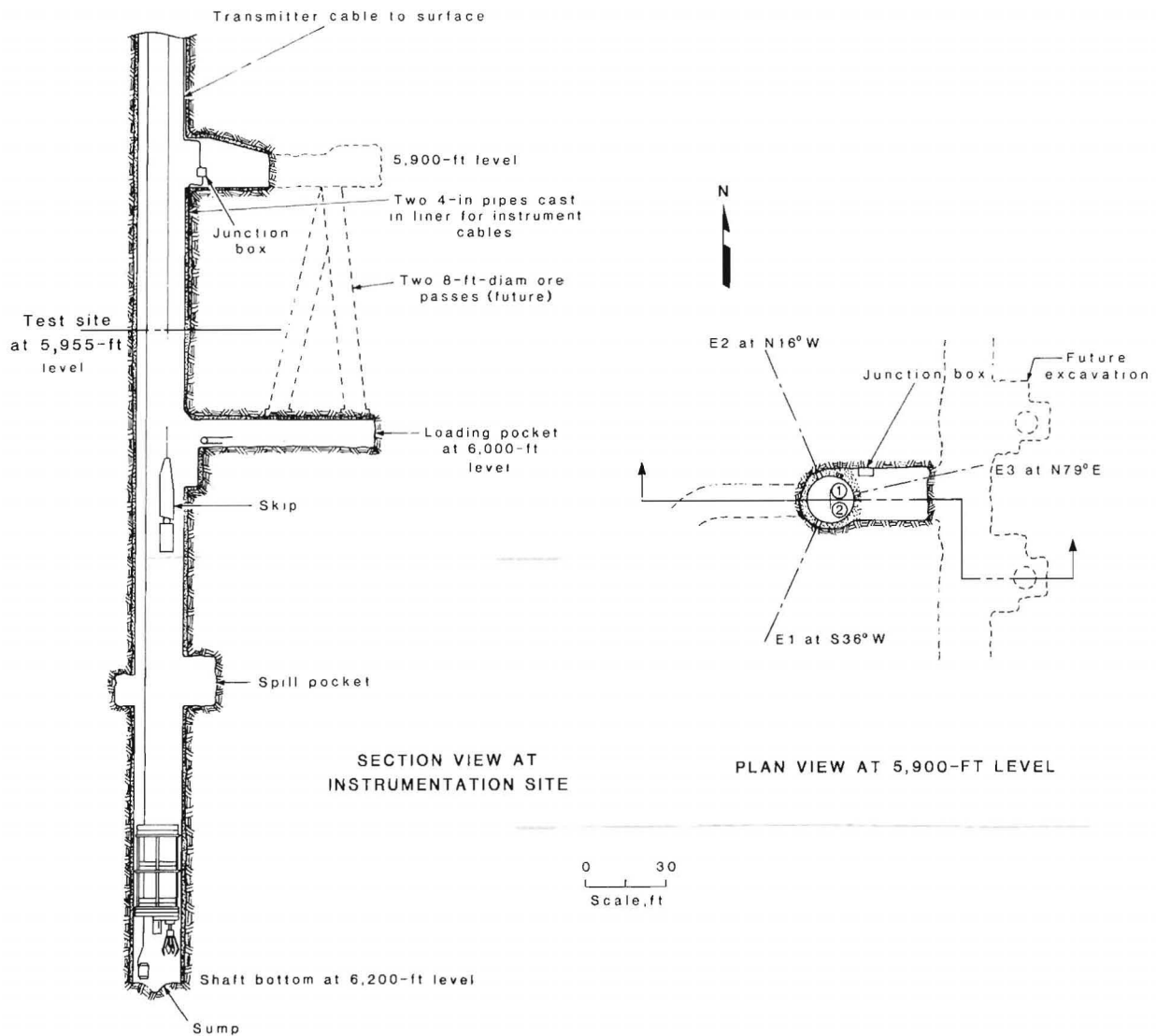


FIGURE 2. - Section and plan views of 5,955-ft test site area.

curb as reference, as the instrumentation was unpacked and prepared. The PC's and SG's were fixed into position midway between the shaft walls and the curb, using reinforcing bar to hold them in place.

The pressure cells were located 90° apart, starting with PC 1 at the first MBPX (E1) position, and oriented to measure tangential stress. SG's were also tangentially oriented and placed next to each corresponding PC. In addition, SG's

were installed at intermediate locations about 45° apart to obtain a strain distribution. The curb-ring pour was completed, the instrument lead wire conduit was tied to the wire mesh and routed through the 4-in pipe to the J-box, and the main forms were lowered for finishing the pour. Installation time for the PC's and SG's was 3 h. Figure 4 shows the final instrument locations at the 5,955-ft level.

RESULTS

From the shaft J-box, data were multiplexed via twisted-pair cable to a microcomputer-based data acquisition system at the surface. Initial data

processing at the test site provided plots of displacement and liner stress or strain versus time. The MPBX's were initialized prior to the first blast. Net

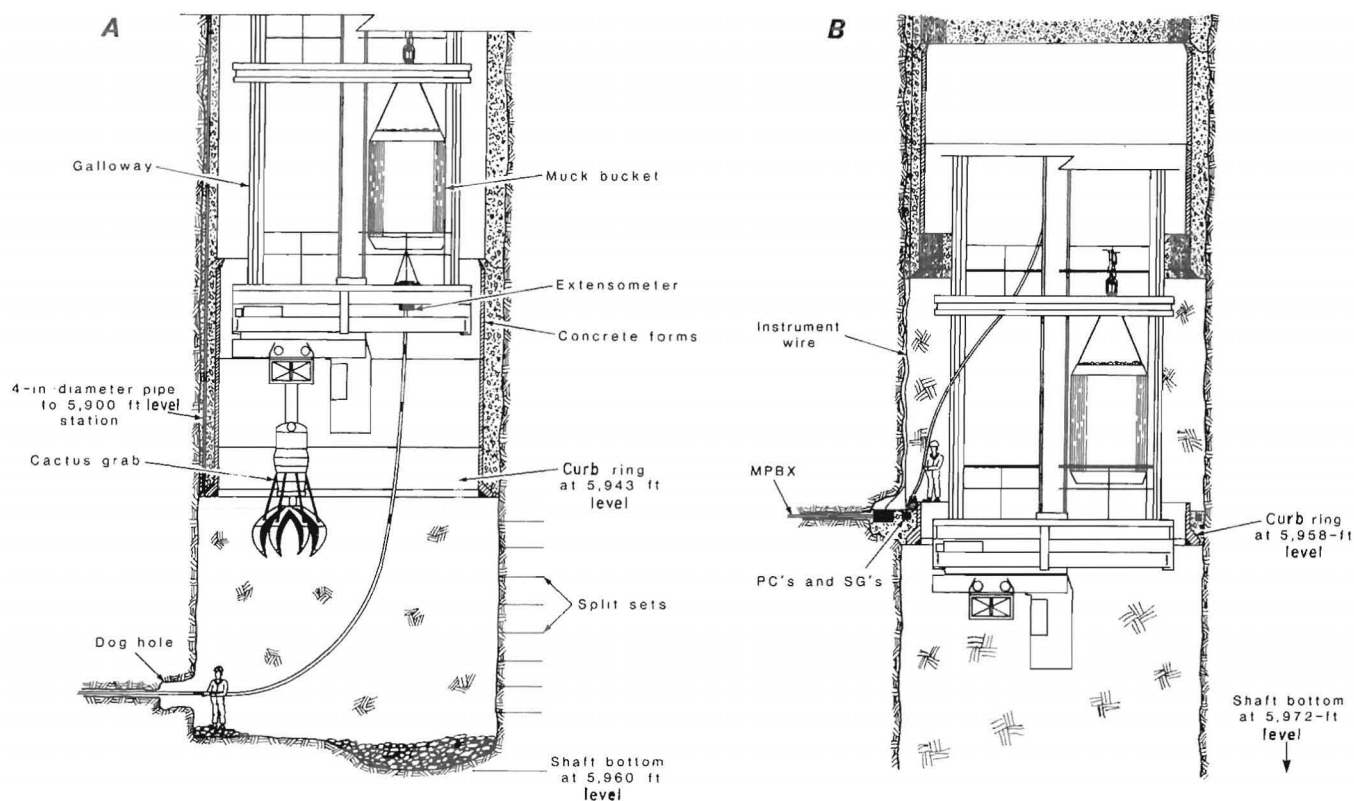


FIGURE 3. - Schematic of techniques used for instrument installation. A, MPBX bundle; B, pressure cells (PC's) and strain gauges (SG's).

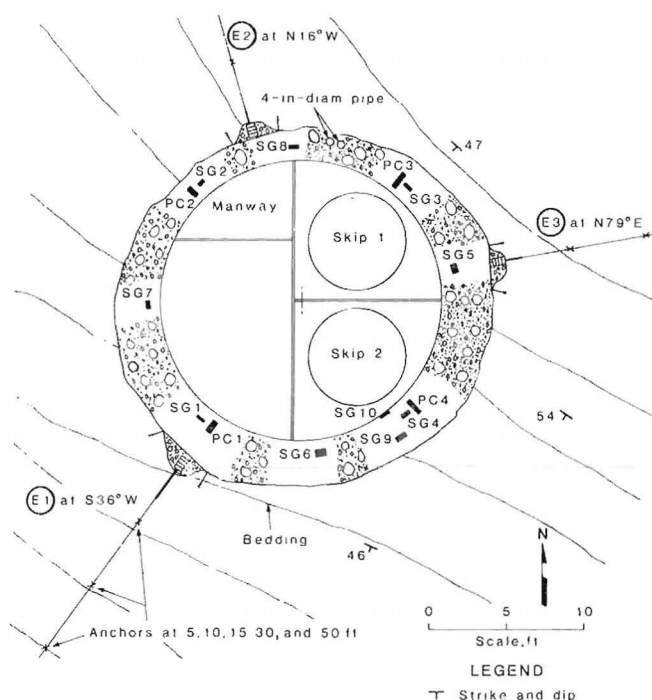


FIGURE 4. - Plan view of the 5,955-ft test site showing final location and orientation of instruments and geologic structure.

displacement of each anchor location is relative to the 50-ft anchor, which was assumed to be stable with zero displacement. Pressure cell and strain-gauge readings stabilized within 24 h after installation in the curb ring, and prior to any significant load buildup in the liner.

ROCK DISPLACEMENT

All of the MPBX's were installed in relatively competent massive quartzite, and as near the shaft bottom as possible. Subsequent bench excavations affect corresponding MPBX's to a greater or lesser degree, depending on the proximity of the bench being excavated. By pulling 6-ft rounds on alternating north and south benches, the average full-face shaft advance was about 3 ft per blast.

Figures 5A, 5B, and 5C show 30-day plots of displacement versus time for MPBX's E1, E2, and E3, respectively, oriented roughly normal, parallel, and 45°

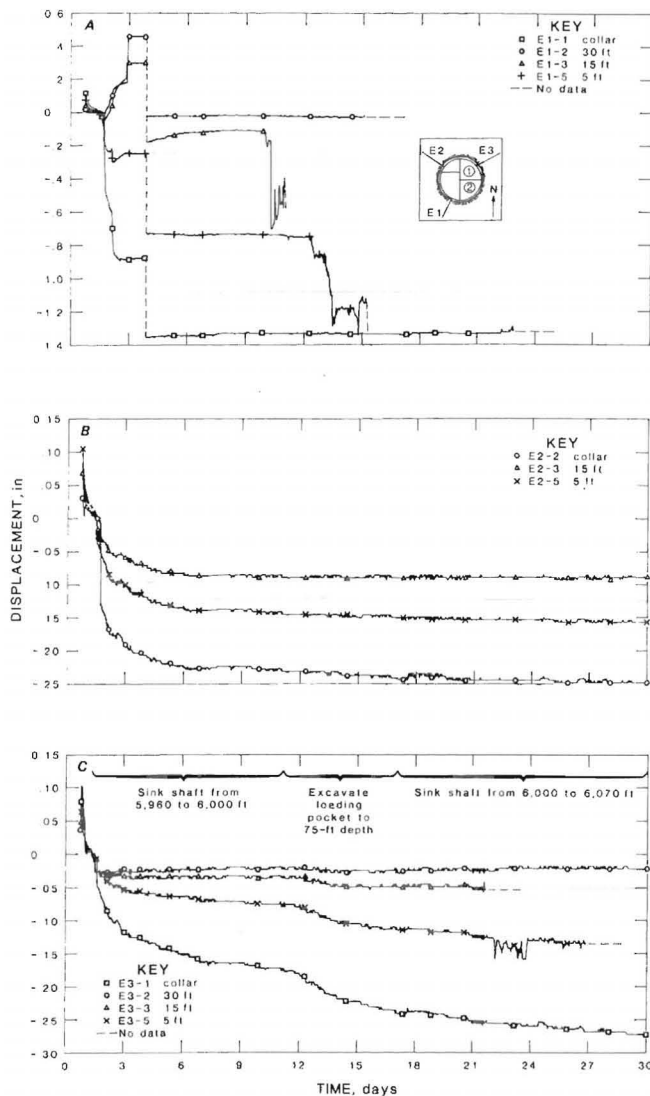


FIGURE 5. - Plot of 30-day displacement versus time. A, MPBX-E1; B, MPBX-E2; C, MPBX-E3.

to the bedding planes, as shown in figure 4. The apparent displacement prior to the initial blast is due to temperature and grout-curing effects on the potentiometers, anchors, and extension rods. However, by the first blast, at 1-1/2 days elapsed time, all of the instruments had reached temperature stabilization.

Initial displacement rates were quite rapid following each blast, indicating elastic response. Successive blasting produced lessening response, and stability of the shaft walls was indicated when the shaft bottom was 40 ft (two shaft diameters) beyond the instrumented level at

day 10, after 12 drill-and-blast cycles. The maximum displacement occurred at the collar on all of the MPBX's, with the effect of anisotropy indicated by the relative difference in displacement between instruments. The time interval from day 3 to day 10 contains the displacement component that causes the initial development of liner pressure.

Displacements were reactivated as excavation of the loading pocket at the 6,000-ft depth was begun. MPBX-E3 (fig. 5C), which is oriented roughly parallel (within 10°) of the longitudinal axis of the loading pocket, is most responsive. The additional increment of collar displacement due to the loading-pocket excavation, and subsequent shaft sinking, totals almost 60 pct of the initial displacement levels at day 30. The displacement is also fairly deep rooted, as even the 15-ft anchor position shows additional movement.

MPBX-E1 (fig. 5A) has apparently malfunctioned by day 4, primarily due to cable damage. Several channels were also lost when flyrock from the initial blast damaged the signal cable. Thirty days after the installation, 5 of the original 15 MPBX channels continued to operate.

LINER STRESS

Pressure cells were located to measure tangential pressure in all four quadrants of the liner, as shown in figure 4. Pressure measured in the liner, as a function of elapsed time, is shown in figures 6A, 6B, and 6C. Figure 6A is a 10-day plot, with the shaft bottom advancing from 5,972 ft to 6,000 ft. The initial 24-h response shows residual effects of concrete curing, shrinkage stresses, and temperature differential. As the concrete stiffens, substantial pressure begins to develop at blast 8, with the shaft bottom 30 ft beyond the instrumentation. Pressure steadily increases during the next five blasts, stabilizing as the shaft bottom reaches 5,994 ft, almost 40 ft beyond the instrumented level. Note the nonuniformity in

stress, with the PC 1 and PC 3 pair attaining an average magnitude of 1,050 psi, almost 2-1/2 times that of the PC 2 and PC 4 pair.

Figure 6B is a 30-day plot showing the increase in liner pressure as a result of excavation of the 6,000-ft loading pocket. This excavation is oriented normal to the shaft centerline and extends 75 ft to the east, midway between PC 3 and PC 4. Liner pressure at these two locations increased substantially as a result of this excavation. Pressure increase

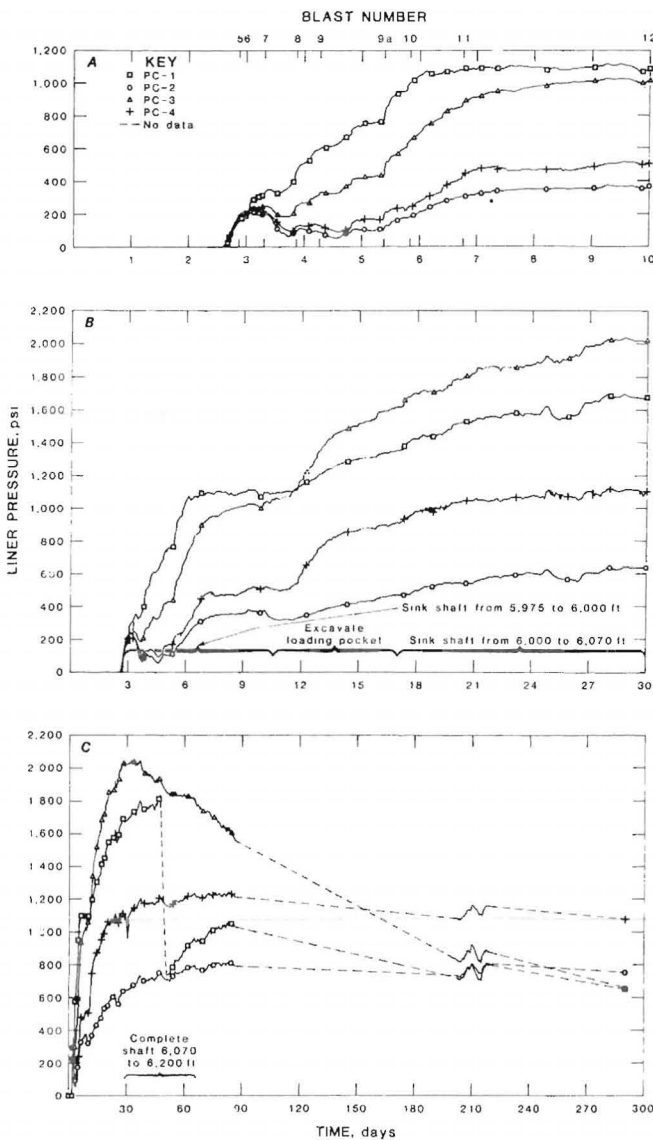


FIGURE 6. + Tangential stress measured in liner as function of elapsed time. A, 10 days; B, 30 days; C, 300 days.

is also evident at the FC 1 and PC 2 locations, although at a much lower rate. Following the loading pocket development, and as the shaft is further deepened from 6,000 to 6,070 ft, pressure continues to increase in the liner at a uniform rate of about 20 psi per day.

Figure 6C shows the complete pressure history up to 300 days since installation. Maximum values of liner pressure are 1,750 and 2,000 psi for PC 1 and PC 3, respectively, attained at about 30 days elapsed time, with the shaft bottom about 120 ft beyond the test site. Peak values of all pressure cells are more than double the initial stable condition reached prior to the loading-pocket excavation. These peaks are supported both by the displacements seen in figures 5B and 5C and by the strain histories seen in figure 7C.

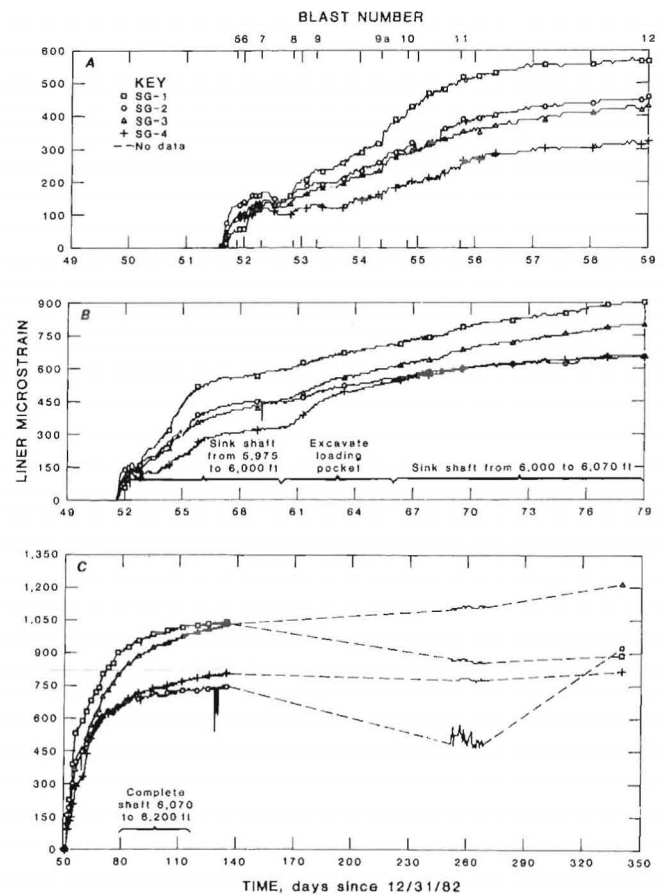


FIGURE 7. + Tangential strain measured in liner as function of elapsed time. A, 10 days; B, 30 days; C, 300 days.

During the completion of the shaft to the 6,200-ft level, PC 1 suddenly dropped to about 50 pct of its peak pressure, partially recovered, and then leveled off at about 30 pct of the peak load. PC 3, also, gradually lost pressure, settling at about 30 pct of the peak value. Pin-hole leaks in the cell body weldment were encountered during calibration and, therefore, are a suspected cause of this pressure drop. Stability of the cells at high pressures is currently being investigated. PC 2 and PC 4 maintain peak levels through shaft completion and up through the latest readings. No data were obtained from day 85 through day 200, and from day 220 to the present time because of signal cable damage. Manual readings being taken indicate uniform liner pressure of about 650 psi (1,100 psi at PC 4).

LINER STRAIN

Strain gauges are located at corresponding pressure-cell locations and at intermediate points, as shown in figure 4. The tangential strain measured in the liner at the PC locations is shown in figures 7A, 7B, and 7C. Figure 7A is a 10-day plot, covering 12 excavation blasts.

As with the pressure cells, initial response during the first 24 h is due to concrete curing and temperature stabilization. Strain increases steadily through blast 11 and levels off as the shaft bottom approaches 5,994 ft at day

10. The grouping of opposite pairs of strain gauges is not as apparent as it was with the pressure cells, although some of this discrepancy may be accounted for by differences in the initial temperature offsets. SG 1 shows the maximum strain of 560×10^{-6} in/in (microstrain), and SG 4 a minimum at 320 microstrain. SG 2 and 3 are at about 450 microstrain.

Figure 7B is the 30-day plot and shows the effects of the 6,000-ft loading-pocket excavation and continued shaft sinking from the 6,000- to the 6,070-ft level. The increase in strain rate is particularly noticeable at SG 3 and SG 4, located on either side of the loading pocket. Strain rates at these locations increased substantially compared to SG 1 and SG 2. Strain continued to increase at a relatively constant rate on all four gauges following the 6,000-ft loading-pocket excavation as the shaft advanced to 6,070 ft.

Peak values were attained on all gauges following completion of the shaft to 6,200 ft at day 60 (fig. 7C). Strain levels remain fairly constant through the latest data scan at day 300; however, several gauges indicate slightly increasing strain. As with the pressure cells, average peak values are more than double the initial stable values attained prior to the 6,000-ft loading pocket. No large strain change is evident in SG 1 and SG 3, corresponding to the PC 1 and PC 3 pressure drop.

PRELIMINARY ANALYSIS

Load mechanisms acting on the shaft structure need to be identified prior to analysis. The most important are (1) in situ stress field, and (2) face advance. Based on the previous rock mechanics work in the district, horizontal stresses are high and unequal, with the largest trending N 30° W to N 70° W. The maximum and minimum stress magnitudes at the 6,000-ft depth approach 9,500 psi and 6,500 psi, respectively.

The effects of face advance have been approximated by three-dimensional numerical modeling.⁵ The model indicates that at a shaft face distance of 8 ft, 80 pct of the radial displacement has already occurred. Therefore, the measured

⁵Schwartz, C. W., and H. H. Einstein. Simplified Analysis for Ground Structure Interaction in Tunneling. MIT Tech. Rep. R79-27, 1979, 426 pp.

component represents only about 20 pct of the total displacement occurring at a given location.

Rock mass displacement around the shaft opening, obviously related to excavation sequence and stress field, also indicates a strong relationship to rock mass anisotropy. Table 1 shows the maximum displacement for each anchor point. The maximum displacement occurs at the collar anchor and decreases rapidly with depth. At the 30-ft anchor depth, displacement is less than 10 pct of that at the collar location and is uniform around the shaft. The initial displacement measured normal to bedding (E1) is about five times that measured 45° to the bedding (E3) and about 3-1/2 times that parallel with bedding (E2). Analysis shows that deformation of this magnitude exceeds an elastic response, suggesting beam bending and buckling of the shaft wall.

TABLE 1. - Maximum radial displacement versus anchor depth, inches

Anchor depth, ft	MPBX and orientation to bedding		
	E1, normal	E2, parallel	¹ E3 45°
Collar..	0.88	0.25	0.17-0.27
5.....	.25	.16	.07- .13
10.....	(2)	(2)	(2)
15.....	.12	.08	.04- .05
30.....	.03	(2)	.03- .03

¹Displacement prior to and following the loading pocket excavation.

²Malfunctioning channels.

Stability is indicated with the shaft bottom 30 to 40 ft beyond the test level, but movement is reinitiated as a result of adjacent excavation outward from the shaft walls, particularly at E3. Displacement of E3, normal to the maximum stress direction, reflects superposition of the effect of the loading pocket and increasing stress concentrations at the intersection of the 6,000-ft loading pocket and the shaft.

The displacement excursions shown by figure 5B and 5C during the 6,000-ft loading-pocket excavations, particularly

the shallow anchors of E3, illustrate the mechanisms for the further stress buildup in the liner and reorientation of the pressure distribution. As the loading pocket is advanced, a displacement bulge develops between the 5,900-ft station and the pocket. This, in turn, increases the liner stresses on either side.

The most striking feature of the liner stress data is the almost immediate pairing of load response (PC1 with PC3, and PC2 with PC4) and the significant liner pressure developed due to the 6,000-ft pocket excavation. Pressure in the northeast-southwest quadrants of the liner is considerably greater, suggesting a maximum stress trending northwest to southeast in the horizontal plane. Initial peak pressure and stability of the shaft liner are indicated as the shaft bottom approaches a 40-ft distance from the test section. The pressure ratio developed at this point, between the northeast-southwest and northwest-southeast pairs, is about 2.5. A simple elastic analysis based on the Kirsch equations shows that a ratio between maximum and minimum applied field stresses of about 1.6 would yield a 2.5 ratio between maximum and minimum liner stresses. This assumes a uniform 1-ft liner thickness with 2 ft of material surrounding the liner, with a reduced modulus about one-tenth that of the intact material. This correlates well with the previously cited projected stress ratio.

Continued sinking after completion of the loading pocket further stresses the liner. As the shaft bottom approaches 115 ft beyond the instrumented level, new peak values are attained of nearly double the initial values, almost 30 days after the liner was poured. Liner pressures 300 days after installation indicate a uniform distribution and final liner equilibrium.

In addition to verifying the pressure cell response, the strain-gauge array can be used to estimate principle axes of tangential strain and, thus, secondary principle stress directions. By applying

a least-squares technique to the strain-gauge data at selected times, an elliptical fit is obtained. Figures 8A, 8B, 8C, and 8D illustrate this fit of the data at elapsed times of 10, 20, 30, and 70 days, respectively, corresponding to significant events in the shaft construction sequence. Data points 1 through 8 are tangential strains measured at strain-gauge locations 1 through 8 (fig. 4), and plotted as vector quantities, with the origin at shaft center. The equation of each ellipse indicates major and minor strain as major and minor (a and b) axes of the ellipse. Figure 8A shows a maximum strain of 532 microstrain and a minimum of 407 microstrain, with the minor strain axis oriented N 48 W, indicating

principle field stresses congruent with this orientation.

During the following 10-day period, the loading pocket is excavated. Figure 8B shows the resultant ellipse generated at day 20. The minor axis has rotated to N 17° W, showing the superimposed effect of the changed geometry. Figure 8C illustrates that the orientation of the stress axes is maintained as shaft sinking commences. The length of the axes, and thus, magnitude of major and minor stresses, also increases significantly from day 20 to day 30. Orientation of the ellipse remains essentially the same through completion of the shaft at an elapsed time of 70 days, although

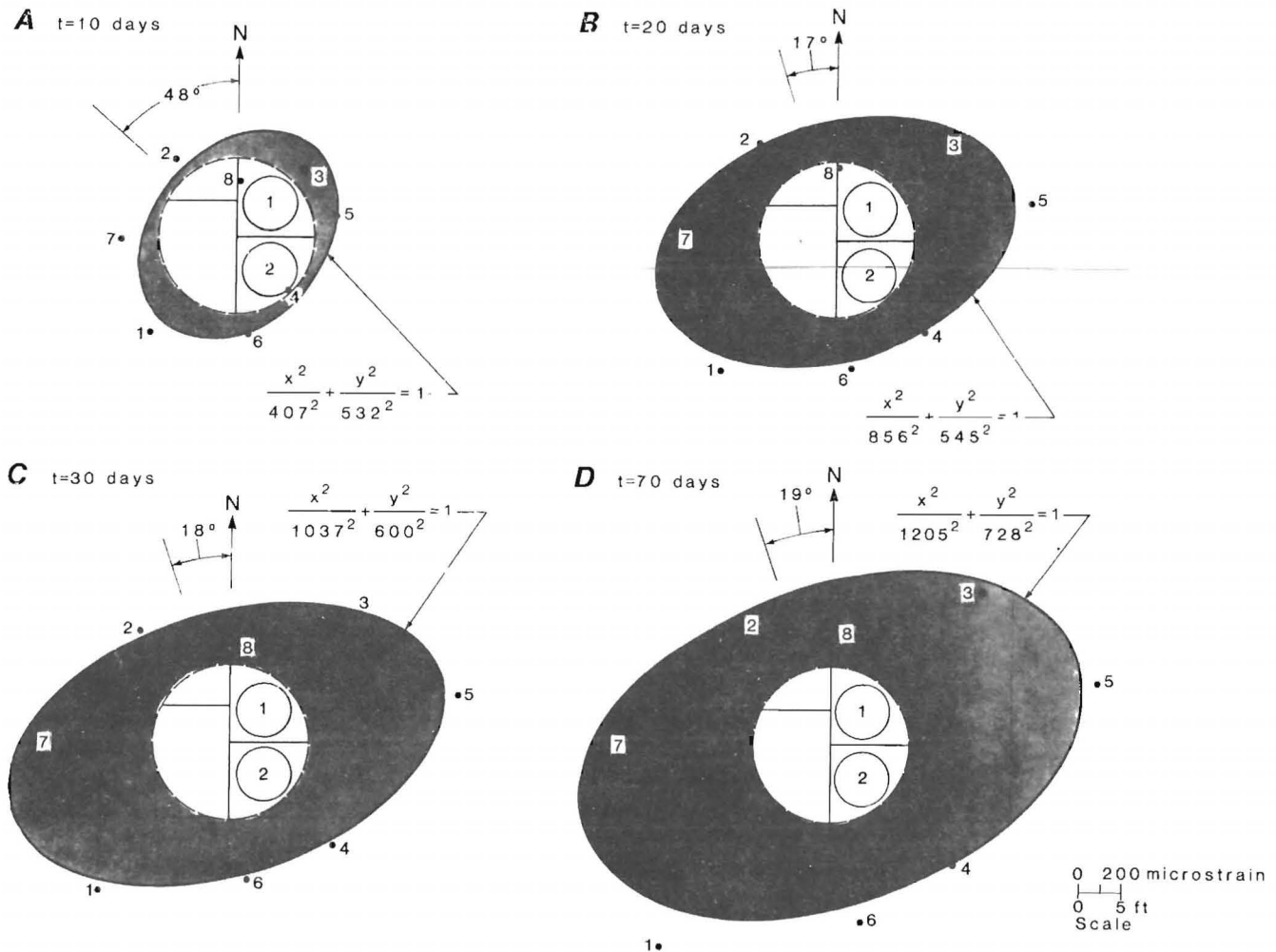


FIGURE 8. - Least-squares elliptical fit of tangential strain for various elapsed times. A, 10 days; B, 20 days; C, 30 days; D, 70 days.

strain magnitudes have steadily increased between 11 pct and 23 pct (fig. 8D). Average deviation of the fitted points ranges from 21 pct (fig. 8A) to 11 pct (fig. 8C).

The preceding analysis precludes a detailed numerical analysis and is very preliminary, but it serves to indicate the potential value of the data as a base for further analyses. Possible approaches include (1) analyzing the strain-gauged liner as a hollow inclusion

stress gauge, with stress relief from shaft advance, i.e., undercoring, (2) converting strain to equivalent stresses, using Hookes Law, (3) determining a concrete system modulus by plotting liner pressure versus corresponding liner strain at selected points on the load path, and (4) determining rock mass modulus distribution by matching observed displacements with those calculated by closed-form analysis. These analyses and others are currently underway, in addition to boundary and finite element work.

CONCLUSIONS

The in situ measurement of rock mass displacement and liner stress and strain at the 5,955-ft level of the Silver Shaft has yielded several significant findings. The steeply dipping bedding planes and the high, unequal stress environment control displacement. Most of the displacement occurs within 5 ft of the shaft walls, degrading rapidly with radial distance. Initial displacement stability is achieved as the shaft bottom reaches 40 ft below the test level. Secondary excavation, such as loading pockets, stations, etc., can be expected to influence rock movement and liner pressure at least as much as does shaft sinking.

The measured liner stresses suggest a strong northwest-southeast trending major stress about 1-1/2 times larger than the minimum stress in the horizontal plane. Initial liner stability is achieved with the shaft bottom about 40 ft distant, or two shaft diameters, correlating with displacement stability and elastic rock response. The loading-pocket excavation caused liner stresses and strains to increase and reorient. Continued shaft sinking following this construction

caused renewed stress and strain response in the liner. Maximum peak stresses occur with the shaft bottom 120 ft from the instrumented section. Visual inspection of the test area of the shaft shows no evidence of surface cracking or failure.

The liner strain gauges yielded excellent data and reflected a nonuniform response. A statistical analysis shows that the major and minor stresses acting on the liner are biaxial, confirming the relative magnitude and orientation of the in situ stress field predicted from previous studies. These data also proved to be a reliable indicator of liner response, particularly as the loading pocket disturbed the initial rock and liner stress equilibrium.

Monitoring of liner strain is considered to be a key element for evaluating liner behavior during future development. The current findings, in conjunction with data from the previous test levels, should provide a valuable data base for understanding the behavior of deep shafts in highly stressed, anisotropic rock masses.